Load Tests on Large Diameter Belled Piles for Rogers Place Arena in Edmonton

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ABSTRACT
Rogers Place, recently completed in the City of Edmonton, Alberta, is one of the most modern arenas in the National Hockey League (NHL) with over 1.1 million square feet of space. The building is supported by 700 belled piles, with the largest bell diameter being 5.4 m, installed within the glacial till deposits underlying the site. Three pile load tests were completed at a range of depths in the till deposit prior to construction to confirm soil conditions, constructability, capacities, and load-displacement characteristics for design. Static top-down test loads up to 14,000 kN were applied, the largest completed in western Canada at the time. This paper describes the pile load test set-up, instrumentation, testing procedure, and interpretation of the results. The results of the load tests confirmed design parameters and enabled an optimized foundation design; the results are compared to the results from other pile load tests from the Edmonton area.

RÉSUMÉ
L'Arena Rogers Place d'Edmonton, en Alberta, est l'une des arénas les plus modernes de la Ligue nationale de hockey avec plus de 1,1 million de pieds carrés d'espace. Le bâtiment est soutenu par 700 pieu a base élargie, avec des pieux ayant un diamètre de 5,4 m. L'argile glaciaire avait une rigidité très variable et contenait des dépôts de sable erratiques. En raison des lourdes charges de fondation et d'un terrain congestionner, des tests de charge ont été essentiels pour optimiser la conception et configurer la disposition des pieux. Pour confirmer les conditions du sol, la constructibilité et les capacités de conception, trois essais de charge de pieux ont été effectués. Les essais ont fourni des informations précieuses sur les caractéristiques de tassement de charge. L'essais statique de haut en bas, un des plus grand test statique complété dans l'ouest Canadien avec une charges appliquées jusqu'à 14 000 KN. Ce document décrit l'installation, l'instrumentation, la procédure de test et l'interprétation des résultats de l'essai de charge sur les pieux. Les résultats des tests ont permis une conception optimisée des fondations ; les résultats se sont comparés à d'autres essais de charge effectuer de la région d'Edmonton.

1 INTRODUCTION

Rogers Place is Edmonton's new arena located in the heart of the ICE District on 104 Avenue between 102 and 104 Street. The arena consists of a seating stadium with a capacity of more than 18,000, an elevated concourse over 104 Avenue, and a community-sized ice rink. The arena has one level of below grade parking. The foundation is composed of more than 700 cast-in-place belled concrete piles with bell diameters up to 5400 mm and factored compressive resistances up to 23,000 kN. Amec Foster Wheeler provided geotechnical engineering, investigation and construction monitoring services on behalf of the City of Edmonton for the arena project between 2012 and 2016.

A phased geotechnical investigation approach was carried out at the site to identify the subsurface conditions at the site and to assess the most suitable foundation type for supporting the loads of the arena structure. It was determined through value engineering that belled piles bearing within the glacial till formation underlying the site would provide a viable foundation option. While local glacial till soils often can provide competent bearing capacity, the issues and challenges for end-bearing piles are the potential disturbance and constructability especially in zones with frequent granular deposits. Constructability issues can be overcome with a high level of contractor workmanship and effective quality control measures, but the requirement for casing deep piles and caving of the bells can make large diameter end-bearing piles costly.

The spatial constraints experienced at the arena site presented a significant challenge of fitting the pile bells within the footprint of the structure. It was necessary to optimize the design. With static load testing, the National Building Code (2010) allows for increasing the geotechnical resistance factor from 0.4 to 0.6 in the calculation of the factored geotechnical resistance of piles, which would result in a reduction in pile sizes.

Due to limited site space and the relatively high structure loads, pile load testing was identified as a early requirement to optimize pile design. The objective of the pile load testing was to confirm the foundation design parameters, establish load deformation characteristics of the piles, evaluate pile constructability, and to permit the use of a higher geotechnical resistance factor. A total of three pile load tests were undertaken in 2013 with applied loads of up to 14,000 kN, the highest top-down test load in western Canada at the time. The test loads were selected to accommodate a large bell diameter that would reflect eventual production piles and minimize scale effects.

This paper presents a summary of the geotechnical investigation carried out, subsurface soil conditions at the site, the pile load tests, and discussion of the test results.

2 INVESTIGATION

2.1 Geology Overview

The geological setting of the site is typical for Downtown Edmonton and has been characterized by Kathol and McPherson (1975). The typical surficial geology in the general area comprises a progression of glaciolacustrine deposits (post-glacial) from 5 to 7 m thick, overlying glacial till deposits extending to depths of 25 to 30 m. The glacial till is mostly composed of clay, silt and sand, and may have saturated sand deposits, cobbles, and boulders. Within the study area, a pre-glacial buried valley incised into bedrock and infilled with fluvial deposits of the Empress Formation underlies the glacial till. The Empress Sands in the area range in thickness from 10 to 15 m. The Horseshoe Canyon Formation bedrock consisting primarily of clay shale and sandstone underlies the project area at a depth of about 40 m below ground surface.

2.2 Subsurface conditions

A site specific geotechnical investigation was carried out to assess the subsurface conditions and to obtain soil parameters required for foundation design. A relatively tight spacing of investigative boreholes was required to assess the variability of the till deposits. Site investigation consisted of conventional drilling techniques utilizing solid and hollow stem augers. Borehole locations are shown in Figure 1. A typical subsurface cross-section is shown in Figure 2. The general stratigraphy of the site consisted of fill soils extending to a depth of 1 to 3 m, over glaciolacustrine clay (i.e. Glacial Lake Edmonton Clay) extending to a depth of 6 to 9 m, over glacial till deposits extending to depths varying between 26 and 30 m. The Empress Sand formation was encountered below the till and extended to the termination depth of the 34 m deep boreholes.

The glaciolacustrine clay was silty, of high plasticity and contained silt laminations and partings. The natural moisture contents of the clay were typically in the 30 to 40 percent range. The SPT "N" values recorded in the clay ranged between 6 and 18 (Figure 3), with the majority of "N" values in the 8 to 15 range. Unconfined compressive strength tests carried out on clay samples indicated undrained shear strengths ranging from 90 to 130 kPa. The reported "N" values and undrained shear strengths were indicative of consistencies varying from firm to very stiff, with the majority of the values indicative of stiff consistencies. At some borehole locations, the lower horizons of the glaciolacustrine deposit at the clay/till interface contained more frequent silt and sand laminations, were generally of medium plasticity, and appeared till-like. The moisture content of these lower horizons in the clay was generally less than the upper zone, and varied between 15 and 30 percent.

The till deposit consisted of silty, sandy, low to medium plastic clay with fine to coarse gravel sizes; scattered coal fragments and random shale nodules; and discontinuous, fine to medium grained silty sand and sandy silt deposits of variable size. The variation of SPT 'N' values with elevation are shown on Figure 3, with SPT 'N' values that exceeded 100 blows per 300 mm of penetration plotted as 100.

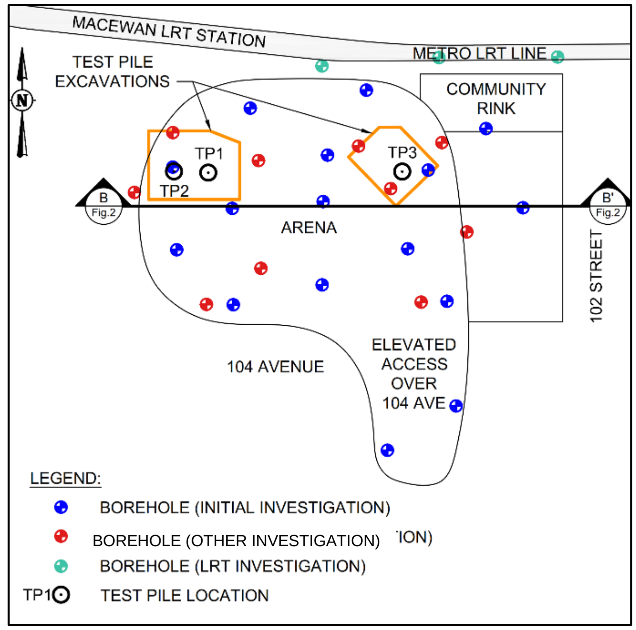


Figure 1. Site plan and borehole locations

Discontinuous deposits of sand, silt or gravel ranging from a few centimetres up to 3 m thick (or more) were encountered within the till. At some boreholes, cohesionless deposits containing thin clay till interbeds were over 6 m in thickness. These discontinuous sand and silt zones within the till were randomly distributed throughout the full thickness of the stratum. Sand deposits were much more common than silt or gravel zones, and were mostly fine grained with vary silt and clay contents. Although the granular deposits were noted less frequently below an elevation of about 650 m and towards the central and southern parts of the site, the distribution, extent and thickness of these intra-till silt/sand/gravel zones were considered to be erratic and unpredictable.

The moisture contents of the clay till typically varied between 13 and 17 percent, which was below or near the plastic limit. SPT 'N' values in the clay till ranged widely, typically from 20 blows per 300 mm of penetration to over 100 blows (Figure 3), indicative of consistencies varying from very stiff to hard. The larger variations in N values, from 20 blows to over 80 blows, were commonly observed in the upper 10 m and also corresponded to the zone of more frequent granular deposits. The majority of SPT "N" values below El. 650 m were in the range of 25 to 45.

The Empress Sand was generally fine grained, silty and contained scattered coal chips. The sand was very dense with typical SPT 'N' values ranging between 78 blows per 300 mm of penetration to 50 blows per 125 mm of penetration. Moisture contents of the sand varied between 7 and 15 percent.

Groundwater level measurements in the standpipes installed in the boreholes indicated the presence of a perched water table within the clay and clay till that was

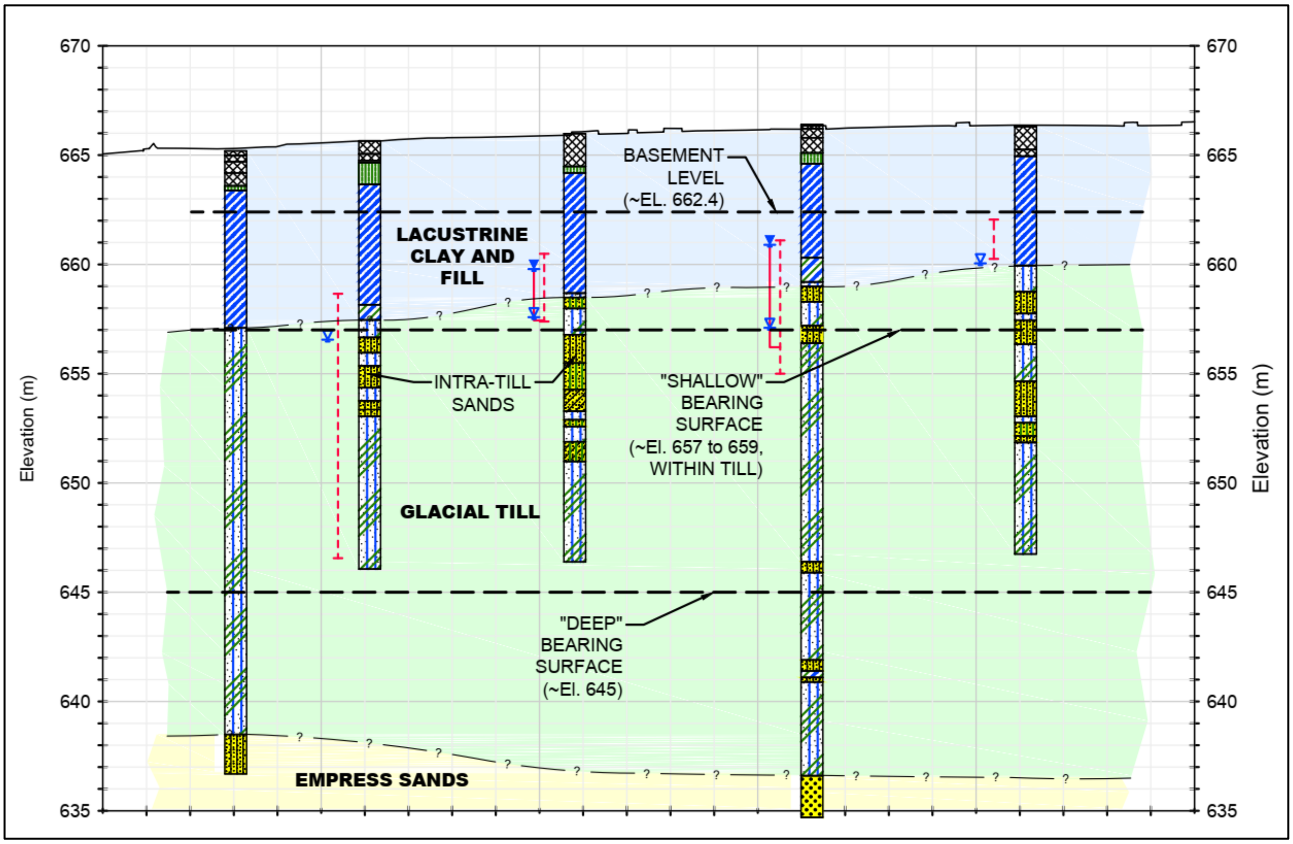


Figure 2. Stratigraphic Cross-section BB' through Rogers Place

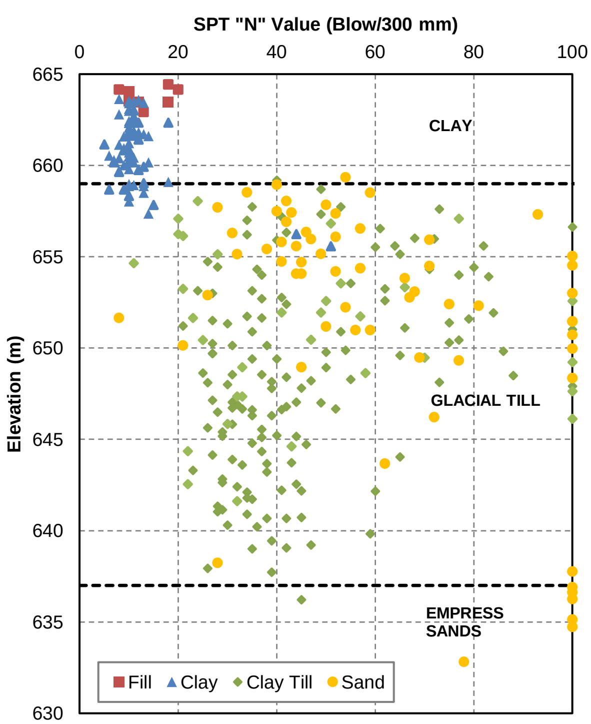


Figure 3. Variation in SPT blow count with elevation

dominated by intra-till sand layers. Observations during drilling indicated that sand layers were often saturated and caving at depths below 5 m (corresponding to ~ El. 661 m). This was an important observation that influenced the design of the pile foundations.

3 FOUNDATION DESIGN APPROACH AND ESTIMATED RESISTANCE

Preliminary design indicated the preferred foundation option was cast-in-place concrete belled piles. It is common practice in the Edmonton area to construct the bells within the hard/dense glacial till deposits, which typically provide good bearing capacities and, when constructed properly, are not prone to excessive settlement. However, the site investigation also indicated the presence of numerous caving and saturated sand deposits, particularly within the upper 5 to 8 m of the till. These deposits represented a constructability challenge for pile excavation and would require extensive casing lengths.

Two bearing stratums were considered for the belled piles (see Figure 2 for schematic representation). The upper bearing stratum was in the shallow till just below the clay-clay till contact and the lower bearing stratum was located deeper (about 12 m below the top of till) within the till. Belled piles founded within the shallow tills minimized the need for extensive casing lengths during pile excavation and were appropriate for lighter compressive loads and where the piles were not required to resist tensile loads. Deeper belled piles were designed to be founded at depths below the majority of the intra-till sands to improve bell constructability. Additional investigation consisting of shallow boreholes up to 12 m deep was conducted following the initial site investigation to further delineate the clay-clay till interface. These boreholes are also shown in Figure 1. This helped to define the “shallow” bearing elevation near the surface of the glacial till which was determined to vary between El. 657 to 659 m. The “deep” bearing elevation was selected at approximately El. 645 m, 12 to 15 m within the till.

Because of the relatively thin cover overlying the shallow belled piles and relatively small depth to bell diameter (z/D) ratios in the order of 1.0, these foundations were designed as circular shallow footings. The deep piles were designed based on end bearing resistance using conventional drilled shaft methods. The design unfactored end bearing resistances for the shallow and deep foundation options were 1100 and 1700 kPa, respectively.

4 PILE LOAD TEST CONFIGURATION

Three pile load tests (identified herein as TP1, TP2, and TP3) were carried out using the top down method in general accordance to ASTM D1143/D1143M-07. Two tests (TP1 and TP3) were carried out on shallow belled piles and one test (TP2) on a deep belled pile. The test sites were selected based on accessibility, existing infrastructure, proposed foundation layout, and representative (average to reasonable lower bound) soil conditions.

Foundation soil conditions similar to those observed in the geotechnical investigation and supplemental boreholes were encountered at the three test pile sites. The base of the bells at the three test sites comprised hard clay till. The consistency of the clay till extending below the base of the shallow belled piles was inferred to vary from very stiff to hard, based on observations from some of the adjacent deeper reaction piles. The upper horizon of the till encountered at the base of TP 1 was of a softer consistency (stiff to very stiff) than expected at the target base elevation. Therefore, Test Pile 1 (shallow bell) was extended about 0.5 m deeper to attain the very stiff to hard conditions in the till.

The load tests were carried out on sacrificial piles (i.e. piles that would not become production piles). The configurations of the test piles and reaction piles were selected to avoid conflict with planned production piles. The layout for TP1 and TP2 relative to the production piles is shown on Figure 4. The configurations of the reaction piles were selected to minimize their installation and loading effects on the test piles. The bases of the reaction piles were typically at 17 m below ground surface. The base of the deep pile (TP2) was approximately 1 m below the bases of the reaction piles. For the shallow test piles (TP1 and TP3), a vertical separation of three bell diameters was maintained between the bases of the reaction and test piles.

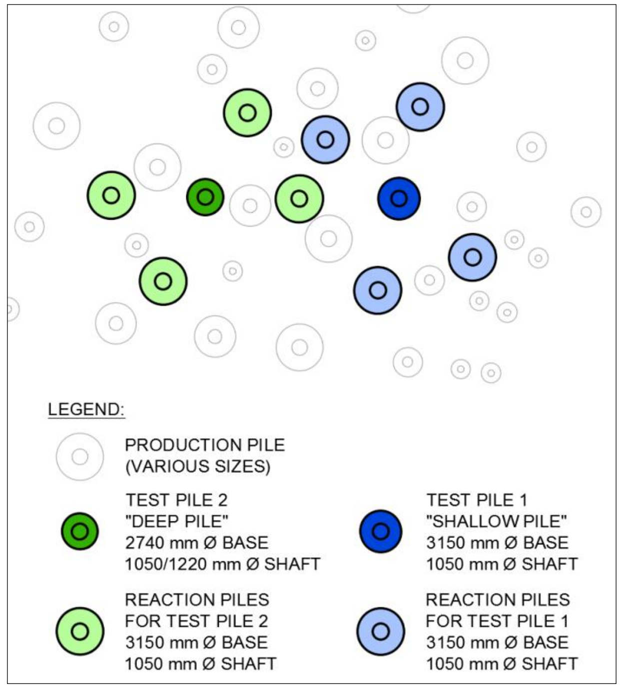


Figure 4. Load test TP1 and TP2 pile layout relative to planned production piles

The load tests were conducted within two large excavations on site as shown in Figure 1. The depths of the excavations were 3 to 4 m below original site grade, at approximately the design elevation of the basement structure (El. 662.4 m). Three test piles were constructed, each with four reaction piles complete with loading frame. The pile shaft diameters varied between 1050 and 1220 mm with bell diameters ranging from 2740 to 3200 mm. Test piles were founded between 5 to 6 m below the excavation for the shallow condition and about 18 m below the excavation for the deep condition. Soil conditions at the test piles, pile dimensions and testing instrumentation are shown in Figure 5.

The test piles were instrumented with electronic strain gauges (GL) and telltales (TT), at different locations over the length of piles. Linear variable differential transformers (LVDTs) and digital micrometers were used to measure displacements at the pile head. The applied load at the pile head was measured with a 20 MN load cell.

The piles were loaded in 700 kN increments, each sustained for approximately 10 minutes (except at design working load) up to the design test load of 14 000 kN. The load interval of 6300 kN (which was taken as the approximate design working load) was held up to 40 minutes for TP1 and 60 minutes for TP2 and TP3.

5 LOAD TEST RESULTS AND INTERPERTATION

Plots showing the applied load vs. measured average pile head displacement for shallow and deep piles are shown in Figure 6 and Figure 8, respectively. Load distributions along the pile shaft for TP1 and TP2 are shown on Figure

7 and Figure 9, respectively. A summary of the pile load test results is provided in Table 1.

Table 1. Pile Load Test Results Summary

Pile Type	Shallow Piles		Deep Pile
Test Pile	TP1	TP3	TP2
Maximum Test Load (kN)	12 600	14 000	14 000
Pile Head Deflection at Maximum Load (mm)	88	65	40
Load at Davisson Offset (Ultimate Pile Capacity)	6600	8900	12 400
Applied Load / Base Area, at Davisson Offset (kPa)	850	1110	2100
Estimated Load Component Due to Shaft Resistance and Pile Cap Effect (kN)	700	800	2000
Estimated End Bearing Pressure at Davison Offset (kPa)	760	1010	1760

At the maximum test load, a clear failure condition had not been reached for all tests. Several load test interpretation methods in the literature were considered. In most cases, conventional load test interpretation methods did not produce a clearly defined ultimate (failure) load or provide a substantial and unrealistic value, exceeding the test load for the piles. Based on CFEM (2006) which summarizes the use of the Davisson (1973) Offset method using the base diameter of the pile, the “ultimate” load was interpreted for each test pile, as shown on Figure 6 and Figure 8. It has been shown by many researchers such as Fellenius (1980) and Prakash & Sharma (1990) that the Davisson Offset method generally results in lower failure loads, and is considered conservative. The Chin (1970) and Brinch-Hansen (1961) failure criteria predicted much higher ultimate pile capacities than the Davisson Offset method, but with corresponding large displacements which exceeded 100 mm.

5.1 Shallow Belled Piles

The applied load-displacement curves for the shallow pile load tests TP1 and TP3 are shown in Figure 6. TP1 and TP3 were loaded to maximum loads of 12 600 kN and 14 000 kN. At the maximum loads, TP1 and TP3 experienced a total pile displacement of 88 mm and 65 mm, respectively. Test TP1 was terminated prior to reaching the specified maximum load due to excessive settlement for the loading frame.

The distribution of mobilized shaft resistance was determined using strain gauge measurements. Figure 7 shows the distribution of resistance for TP1 as an example. The length of the shaft above the shallow bells was only about 3 m, and thus about 90 percent of applied load was borne by resistance provided at the pile base.

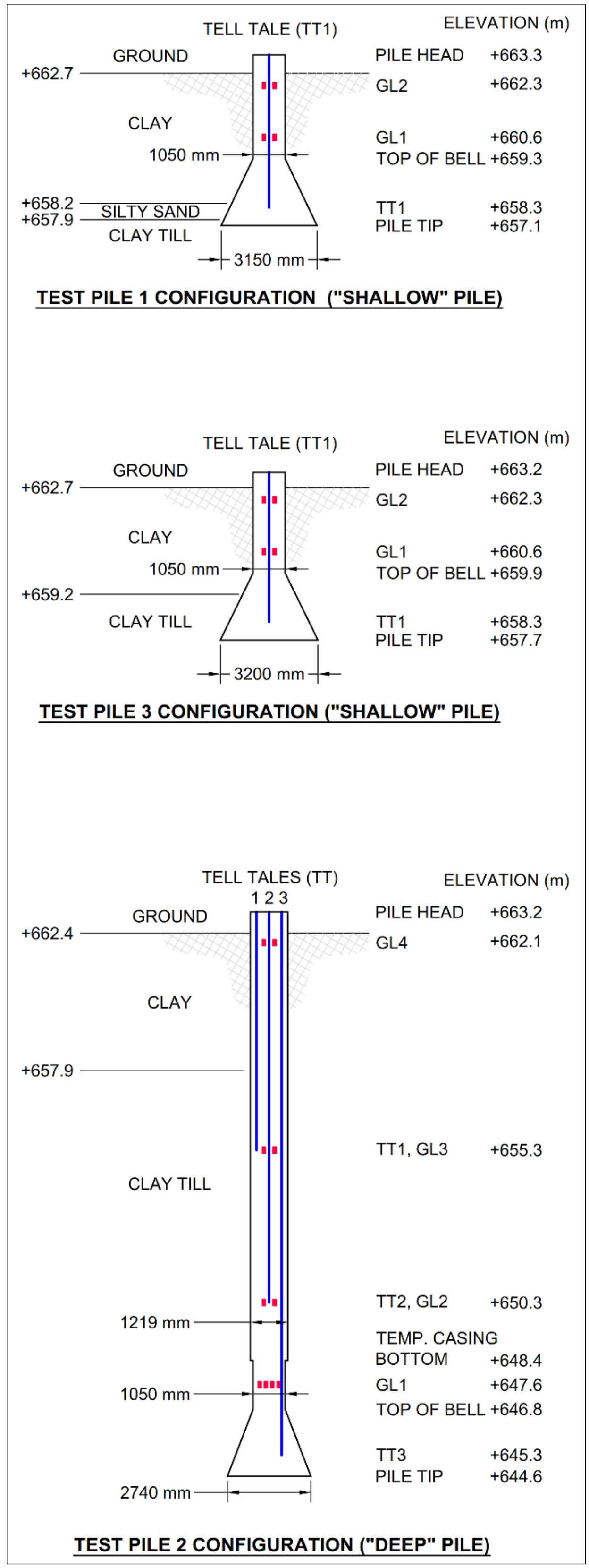


Figure 5. Schematic section of test piles

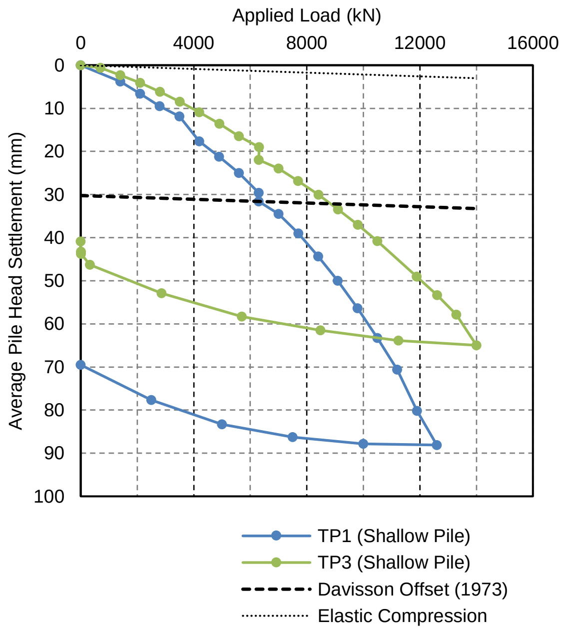


Figure 6. Load displacement curve for shallow pile load tests TP1 and TP3

The ultimate loads predicted by the Davisson method for TP1 and TP3 were 6600 and 8900 kN, respectively. Subtracting the component of the loads attributed to shaft resistance, the ultimate end bearing resistances were determined to be 760 and 1010 kPa for TP1 and TP3.

The softer response of test pile TP1 was attributed to natural variation in soil conditions across the site as well as potential base disturbance during belling. The average SPT 'N' values inferred below the pile bells, from the boreholes near each test pile, were comparable, in the range of 33 to 36. It is possible that the pile was founded on some disturbed sand that may have caved in during test pile construction and concrete placement. Silty sand was observed at the bell cavity as shown on Figure 5. The difference in pile displacement response was deemed not to be the result of differences in elastic shortening of the pile due to similar concrete strengths between TP1 and TP3.

5.2 Deep Belled Piles

The applied load displacement curve for the deep pile load test, TP2, is shown in Figure 8. At the maximum test load of 14 000 kN, a pile head settlement of 40 mm was observed. Similar to the shallow pile tests, no plunging failure was observed.

The distribution of resistance from strain gauge measurement with depth is shown in Figure 9. Approximately 85 percent of the resistance was provided by the pile base.

Applying the Davisson Offset method to test TP2 and subtracting the shaft resistance component, the ultimate end bearing resistance was determined as 1760 kPa.

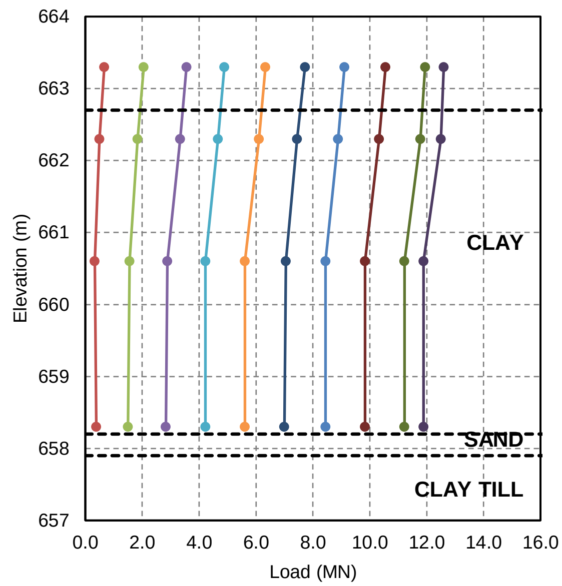


Figure 7. Load distribution along pile length, at different load increments, for shallow pile load test TP1

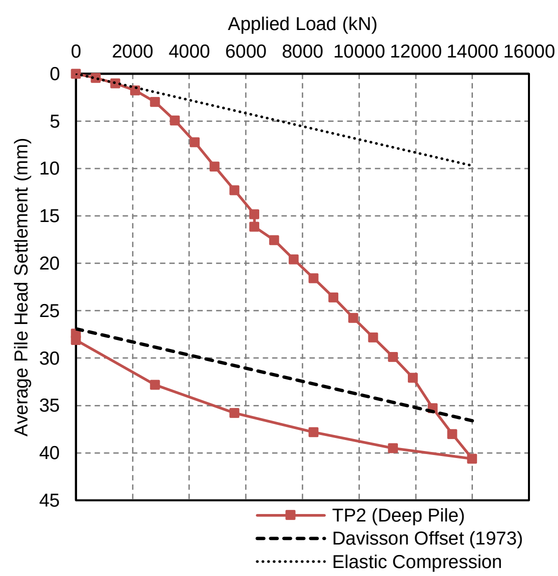


Figure 8. Load displacement curve for deep pile load test

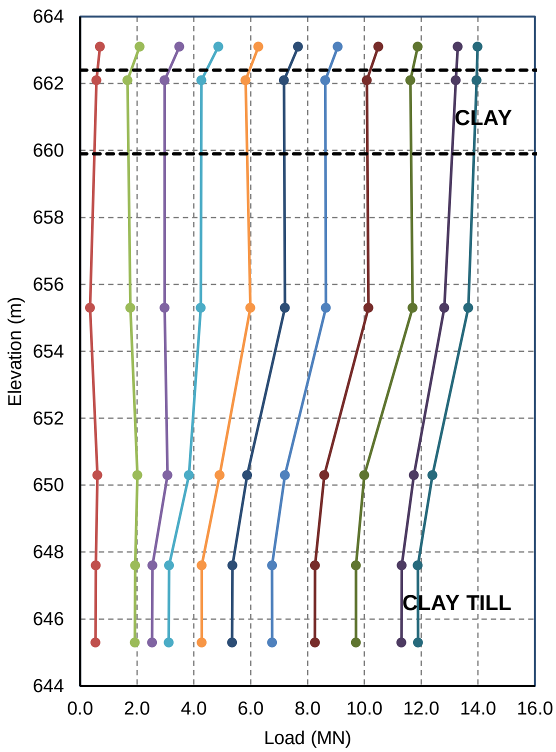


Figure 9. Load distribution along pile length, at different load increments, for deep pile load test TP2

6 DISCUSSION

The pile load test results allowed confirmation of design parameters, use of a higher geotechnical resistance factor, and optimization between shallow and deeper pile layout. Field observations during the installation of the test and reaction piles provided insight into behavior of soils at the site during pile installation. These observations were valuable in developing the quality assurance program for production piles. Observations made from the load tests and installation of production piles are discussed in the following sections.

6.1 Comparison with Design Parameters

In general, there was good agreement between the design geotechnical resistances and the observed capacities and load-settlement characteristics of the test piles. The response of the test piles under the applied test loads showed some variability, which confirms a range in soil conditions as encountered during geotechnical investigation at the site.

As expected, the resistance of the test piles was substantially dictated by end-bearing. A frictional resistance component, albeit small, was observed during the pile load tests and would be expected with production belled piles. The contribution to the pile resistance will vary from pile to pile depending on length of shaft, the presence of sand layers and sloughing in the till and the general conditions at the time of construction.

In the absence of clearly defined ultimate (failure) load, especially in the case of shallow piles, and the expected large deformation required to achieve such load for large diameter belled piles, the design resistance is best determined based on deformations under service loads. For the deep pile, the test results confirmed the design end bearing resistance predicted from soil parameters. Due to the presence of thick sand deposits within the till and the potential of sloughing and caving during pile installation, side shear friction along the shaft was neglected in the design. For the shallow piles, the ultimate load interpreted from TP3 was considered more representative of typical soil conditions at the site. More importantly, the pile head settlement under service load (equal to 60 to 70 percent of the factored geotechnical resistance) was 12 to 14 mm for TP 1 and 8 to 9 mm for TP3, which was considered acceptable.

In view of the load test results, it was concluded that belled piles could be designed based on ultimate bearing resistances provided for shallow and deep belled piles. A summary of design parameters is presented in Table 2.

Table 2. Design Parameters Summary

Pile Type	Shallow Piles	Deep Pile
Ultimate Bearing Resistance (kPa)	1100	1700
Factored Bearing Resistance (Using a Geotechnical Resistance Factor of 0.6) (kPa)	660	1020
Pile Head Deflection from Load Test at Serviceability Load (mm)	8-14	5-8
Ratio of Pile Head Deflection from Load Test at Serviceability Load to Bell Diameter (%)	0.3 – 0.4	0.3

6.2 Load Test Setup

Although the load test was successful, a few challenges were encountered largely arising from the relatively small site space, high density of foundation elements and the relatively high test loads specified.

Four jacks were required to provide the maximum specified load of 14 000 kN. ASTM D1143 recommends the use of a single jack, which resulted in some eccentric loading. Based on measurements taken during and following the dismantling of the load tests, it was found that the centre of the applied load varied from 5 mm to 30 mm of the center of the test pile, with TP 1 being the highest.

6.3 Constructability

Accurate delineation of the clay-clay till contact was essential to the success of the shallow belled piles. The depth of the clay-clay till contact varied across the site by 2 to 3 m and experienced geotechnical personnel provided full-time monitoring during pile construction to confirm that the shallow bells were founded on the competent tills rather than the “till-like” clay commonly found at the surface of the till. During construction of production piles, care was exercised to confirm that the base was seated in competent till soils and that bases were thoroughly cleaned prior to pouring concrete. For deeper piles, down-hole camera inspection was performed in selected piles.

There was a small risk of the belled pile excavations collapsing due to the thin cover of comparatively softer clay overlying the tills and the large bell diameters. It was identified at the onset of the project that a crane type piling rig may be required so that pile excavation would not undermine the rig during bell excavation. During construction, the length of the pile shafts above the bell as short as 1 to 2 m with bell diameters of up to 4 m were successfully constructed using conventional piling rigs.

Of the 473 piles designed as "shallow" piles, 51 piles were ultimately extended between 3 and 17 m beyond their design depths. In half of these cases, extensions in the order of 3 m were required to achieve sufficiently stable excavations to construct the pile bell. Pile excavation stability was predominantly dictated by the ingress of groundwater from saturated sand deposits within the till, some of which were several metres thick.

7 COMPARISON WITH OTHER BELLED PILE LOAD TESTS IN EDMONTON TILLS

In 2016, Amec Foster Wheeler conducted a series of pile load tests for the Kathleen Andrews Transit Garage (formerly known as the North-East Transit Garage or NETG) near the interchange of Yellowhead Trail and Fort Road for the City of Edmonton. The NETG is located approximately 6 km northeast of the Arena. The pile load tests were conducted to verify the design parameters and to permit the use of a higher geotechnical resistance factor. The details of the load tests were presented in Amec Foster Wheeler (2016) Report.

The subsurface conditions at NETG were generally similar to those of the Arena. The general stratigraphy consisted of about 4 to 5 m of fill over about 1 m of glaciolacustrine clay over glacial till. The till encountered at the NETG was of a very stiff consistency with typical SPT 'N' values of 25 to 30, below the base elevation of the test piles. Variable thicknesses of dry to saturated intra-till sands were present throughout the site. Belled cast-in-place concrete piles were also the preferred foundation option.

Three pile load tests, conducted in a similar fashion to those at the Arena, were conducted on belled piles founded at depths between 7 to 10 m below the ground surface or 5 to 7 m into the till. The test piles had bell diameters of 2.1 or 2.7 m and were subjected to test loads of up to 7000 kN. Ultimate bearing resistance for the pile bells founded in clay till were found to vary between 1200 and 1750 kPa. The end bearing typically contributed 75 to 90 % of the total pile resistance.

Published results from pile load tests in Edmonton till (Wang et al. (2015); Ruban and Kort, (2011); and Amec Foster Wheeler's predecessor Hardy Associates) were compiled and compared to evaluate correlations with the measured bearing resistances.

For compressive loads, the ultimate axial pile resistance is calculated as the summation of both skin friction along the pile shaft and end-bearing resistance at the base. It is customary in local practice to use the total stress method as per CFEM (2006) to calculate pile resistance in clay till. As discussed above, the contribution of side friction for belled piles is relatively small and the belled piles derive most of their resistance from end bearing. Therefore, the discussion below is limited to end bearing resistance.

The end bearing resistance in cohesive materials is calculated using the total stress method suggested in CFEM (2006), which is presented by Equation 1.

$$q_t = N_t S_u \qquad [1]$$

where:

q_t = unit end-bearing resistance
N_t = bearing capacity factor
= 9 (for pile diameter smaller than 0.5 m)
= 7 (for pile diameter of 0.5 to 1 m)
= 6 (for pile diameter larger than 1 m)
S_u = undrained shear strength of soil

Where the values of S_u were not provided in the reference, the following local empirical correlation was used to estimate the undrained shear strength of the soil.

$$S_u = 5.5\ N \qquad [2]$$

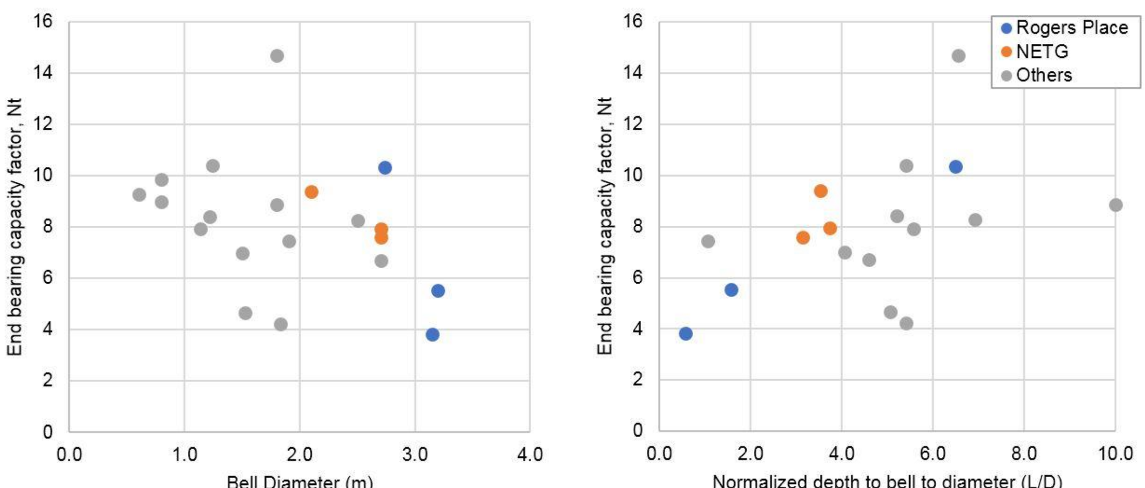
where:

N is the SPT number of blows per 300 mm of penetration within two bell diameters below the pile base.

A summary of the pile load test results is presented in Table 3. The back calculated bearing capacity factors are shown in the table. Figure 10 (a and b) show the variation of N_t values with bell diameter (D) and the pile embedment ratio (L/D). For L/D less than 3, the reported N_t values varied between about 4 and 7.5 with an average value of 6, which agrees with the bearing capacity factor for shallow foundation design. For L/D greater than 3, N_t values varied considerably with majority of the results between 7 and 10. No discernible correlation was observed between N_t and the bell diameter. The range would broadly reflect the inherent heterogeneity of the glacial till deposits. It is acknowledged that the criteria used to define the ultimate resistance will affect the calculated N_t values. Examination of Table 3 shows that the 7 to 10 range was predicted by the various criteria used.

Table 3. Summary of interpreted bearing capacity factors (N_t)

Test Location	Ultimate Load Calculation Method	Shaft Diameter (m)	Bell Diameter (m)	Pile Embedment Depth (m)	N_t	Reference
Rogers Place Arena TP1	Davisson Offset	1.1	3.2	5.6	3.8	AMEC (2013)
Rogers Place Arena TP3	Davisson Offset	1.1	3.2	5.0	5.5	AMEC (2013)
Rogers Place Arena TP2	Davisson Offset	1.1	2.7	17.8	10.3	AMEC (2013)
Transit Garage TP1	Davisson Offset	0.8	2.1	7.4	9.4	AmecFW (2016)
Transit Garage TP2	Maximum Test Load	1.0	2.7	8.5	7.6	AmecFW (2016)
Transit Garage TP3	Davisson Offset	1.0	2.7	10.1	7.9	AmecFW (2016)
Santa Rose No.1	Excessive Settlement	0.8	0.8	20.1	9.0	Thomson (1980)
Santa Rose No.2	Excessive Settlement	0.8	0.8	18.3	9.9	Thomson (1980)
Santa Rose No.3	Excessive Settlement	0.6	0.6	14.3	9.3	Thomson (1980)
Bio. Sci. Bldg.	Excessive Settlement	0.8	1.2	6.7	10.4	Thomson (1980)
Kaskitayo A	Excessive Settlement	0.4	1.1	6.4	7.9	Thomson (1980)
Kaskitayo B	Excessive Settlement	0.4	1.2	6.4	8.4	Thomson (1980)
Strathcona	Excessive Settlement	0.9	1.9	2.0	7.5	Thomson (1980)
Anthony Henday Drive and 127 St	Maximum Test Load	0.9	1.8	18.0	8.9	Ruban and Kort (2011)
97 St and 104 Ave	Maximum Test Load	1.0	1.8	11.8	14.7	Wang et al. (2015)
112 St and 104 Ave	Maximum Test Load	1.0	2.7	12.4	6.7	Wang et al. (2015)
103 St and 108 Ave	Maximum Test Load	1.0	2.5	17.3	8.3	Wang et al. (2015)
170 St and 87 Ave	Excessive Settlement	0.6	1.8	9.9	4.2	AmecFW Internal Files
111 St and 51 Ave	Excessive Settlement	0.6	1.5	7.7	4.7	AmecFW Internal Files
17 St and 92 Ave	Excessive Settlement	0.6	1.5	6.1	7.0	AmecFW Internal Files



(a) (b)

Figure 10. Variation of bearing capacity parameter (Nt) with pile bell diameter (a); and Pile embedment ratio (b)

8 CONCLUSION

Pile load tests were carried out on concrete belled piles at the Edmonton Downtown Area site. The test results confirmed the design parameters for production piles. Observations carried out during the installation of test and reaction piles confirmed the feasibility of shallow belled pile construction.

Conducting pile load tests at the site allowed the structural designer to use a geotechnical resistance factor of 0.6 which is 50 % higher than the 0.4 used with design based on semi empirical analysis. The higher resistance factor resulted in design efficiency and reduced the size and cost of the foundations.

It is common in local practice to use a bearing capacity factor of 9 in the design of belled piles in clay till. The results of pile load tests in Edmonton clay till indicated that the bearing capacity factor for piles (L/D ≥ 3) generally varied between 7 and 10, which is a relatively wide range. Pile load tests allows for determining the actual value for a specific site.

Pile load tests allowed for establishing the load deformation characteristics of piles within a specific soil formation. Deformation under service loads can be established from the tests and incorporated into the pile design.

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